APPENDIX H
Geotechnical Report

REPORT

GEOTECHNICAL REPORT

MUSCOY PLUME REMEDIAL DESIGN PROJECT I-215 AND BNSF UNDERCROSSING

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1.1 PROJECT DESCRIPTION

This report presents the results of our geotechnical investigation for design of an approximately 255-foot long pipeline undercrossing of I-215 and the BNSF Railroad in San Bernardino, California. As part of the Muscoy Plume Remedial Design Project, the Environmental Protection Agency (EPA) is planning to construct this pipeline to convey contaminated water from the Muscoy Groundwater Contamination Superfund Site to a treatment plant. Trenchless construction methods will be required to cross under I-215 and the BNSF railroad tracks in the vicinity of 10th Street. This element of the overall project focuses on the pipeline crossing segment between Station 145+65 and 148+20. Figure 1 provides a general vicinity map for the project.

Construction will involve installation of a 32-inch diameter steel casing followed by a 20-inch diameter internal carrier pipe. A maximum pipeline elevation of 1092 feet has been set to allow for future utility construction within Caltrans right-of-way. This requirement will provide approximately 12 feet of cover above the pipe. The pipes will be connected to proposed cut and cover pipeline segments to the east and west. Inclined (transition) pipe segments will be constructed east and west of the trenchless undercrossing to achieve reduced depths outside of the Caltrans corridor.

1.2 PURPOSE AND SCOPE OF WORK

The purpose of this investigation was to evaluate subsurface conditions along the proposed pipeline undercrossing alignment to facilitate development of geotechnical criteria for pipeline design and construction. This report provides the following:

- Discussion of the field exploration and laboratory testing programs;
- Summary of the laboratory test data;
- Description of the site and subsurface conditions encountered during the field exploration program;
- Evaluation of anticipated subsurface conditions during construction;
- Evaluation of trenchless construction alternatives;
- Recommendations for a preferred trenchless construction method;
- Evaluation and recommendations for jacking and receiving pits; and
- Summary of major findings and conclusions.

1.3 REPORT ORGANIZATION

This report is organized as follows:

- Section 1: Introduction
- Section 2: Field Exploration and Laboratory Testing Program
- Section 3: Alignment Surface and Subsurface Conditions

- Section 4: Trenchless Construction Evaluation
- Section 5: Summary and Conclusions
- Section 6: Limitations
- Section 7: References
- Appendix A: Field Exploration Program
- Appendix B: Laboratory Testing Program

2.1 EXPLORATORY BORINGS

Two exploratory borings, numbered B-1 and B-2, were drilled for the proposed undercrossing. The locations are shown on Figure 2, Plan and Profile. Borings B-1 and B-2 were drilled to depths of 40 and 30.5 feet, respectively. The borings were drilled by Cal Pac Drilling using a truck-mounted Mobile B-61 drilling rig. Hollow-stem augers were employed to advance the boreholes. The soil cuttings and samples were logged in the field and soils were visually classified as the drilling proceeded. Samples of the subsurface materials were obtained at selected depths in the borings using two types of samplers; a 2.5-inch O.D. modified California sampler and a 2-inch O.D. SPT split-spoon sampler as shown in the Key to Log of Borings (Figure A-1 in Appendix A). A bulk sample was also taken near the surface of Boring B-1. The samples were returned to our geotechnical laboratory for further visual examination and laboratory testing. Logs of Borings were prepared based on the field logs, visual examination of soil samples in the laboratory, and the laboratory testing results. Detailed logs of borings are presented in Figures A-2 and A-3. Detailed descriptions of the procedures employed to drill the borings, and to obtain soil samples are provided in Appendix A (Field Exploration Program).

2.2 GROUNDWATER OBSERVATION WELLS

Groundwater observations wells were installed in both of the borings. The wells consisted of 2-inch-diameter PVC well casings with well screens installed in the borings. The annular space between the soil and the screened portion of the pipe was backfilled with filter sand. The screened interval and sand pack are located from 18 feet below the surface to the bottom of the hole in both borings. To inhibit surface water infiltration, the upper part of the wells were completed with hydrated bentonite chips and a hydrated granular bentonite seal above the sand pack. Steel traffic-rated protective covers complete the observation wells at the ground surface. A more complete description of the observation well construction details is provided in Appendix A and on the logs of borings.

2.3 LABORATORY TESTING

Representative soil samples obtained from the exploratory borings were tested in our Pleasant Hill geotechnical laboratory in order to evaluate their engineering properties for use in the analyses. The following laboratory tests were performed on selected soil samples:

- Water content (ASTM D2216)
- Dry density (ASTM D2850)
- Unconfined compressive strength (ASTM D2166)
- Atterberg limits (ASTM D4318)
- Grain size and hydrometer analyses (ASTM D422)

The results of the laboratory tests are summarized in the logs of borings at the corresponding sample depths. Detailed laboratory test results are presented in Appendix B (Laboratory Testing Program). Table 1 summarizes the laboratory data.

3.1 SURFACE CONDITIONS

The proposed pipeline crossing is located within public right-of-way near Interstate 215. Access pits will be positioned at the perimeter of residential streets. The proposed vertical alignment passes approximately 13 feet below the BNSF railroad tracks and 16 feet below the Interstate 215 embankment crest. Figure 2 provides a plan and profile view of the proposed crossing as it relates to existing surface features.

3.2 REGIONAL GEOLOGIC CONDITIONS

The proposed tunnel alignment lies within California's Transverse Range physiographic province near the border with the Peninsular Range physiographic province. The Transverse Range province is characterized by an east-west trending geologic fabric that is expressed topographically as east-west trending mountains. This is an anomalous trend that contrasts with the NW-SE direction of the Coast Ranges and the Peninsular Range. The Peninsular Range province is comprised of a series of ranges separated by longitudinal valleys subparallel to faults branching from the San Andreas Fault. The trend of topography is similar to the Coast Ranges, but the geology is more like the Sierra Nevada, with granitic rock intruding the older metamorphic rocks. The tunnel alignment site is located within a valley bounded by the San Bernardino Mountains to the North East and the San Gabriel Mountains to the North West. The valley is filled with Quaternary alluvium (Qal) consisting of sands, silts, and clays.

3.3 SEISMIC SOURCES AND LOCAL FAULTING

The project area is in a tectonically and seismically active area. Figure 1 shows the location of the site relative to the local Quaternary faulting pattern. The study area is dissected by active faults, including the two most significant active faults in southern California, namely, the San Andreas Fault and the San Jacinto Fault. The presence of these active faults produces a relatively high level of seismic activity. The tunnel crossing alignment is located approximately 1.7-miles northeast of the San Jacinto fault. The San Andreas fault is located approximately 4.6 miles northeast of the site. These two faults intersect approximately 10 miles to the northwest of the site. The State of California has designated Alquist-Priolo Earthquake Fault Zones (A-P Zones) along each of these faults. The site is located 1.6 miles outside the nearest A-P Zone; hence, the potential for surface fault rupture at the site is considered very low.

3.4 SUBSURFACE CONDITIONS

3.4.1 Soil

The generalized subsurface profile at the undercrossing consists of silty sand fill overlying interbedded clay, silt, and silty to clayey sands of alluvial origin. Figure 2 provides a profile view showing graphic boring logs with respect to existing site surface features and the proposed pipeline horizon.

Fill materials at both boring locations were characterized as medium dense sand. Thickness ranged between 2 (B-2) and 3 feet (B-1). A thickened fill section is present at I-215, which is positioned on an approximately 4-foot high embankment.

The natural alluvial soil profile consists of a clay/silt layer overlying a silty sand that contains apparent discontinuous clayey sand and sandy clay layers.

The upper clay/silt layer ranged between 7.5 (B-1) and 8.5 feet (B-2) thick. An upper 5-foot thick silt layer was present only at B-2 (west side). This silt was described as dense to medium dense and contained clay. Measured water content, dry unit weight and unconfined compressive strength on a select sample were 23 percent, 102 pcf and 3.5 ksf, respectively. The clay component of the clay/silt layer ranged between 3.5 (B-2) and 5 feet (B-1) thick. This material was characterized as very stiff to hard and of low plasticity. Moisture content, dry unit weight and compressive strength for a select sample were measured as 20 percent, 107 pcf and 6.1 ksf. Liquid and plastic limits were 33 and 19 percent, respectively. A thin (2.5 feet) silt was detected in both borings at the base of the upper clay/silt layer. The silt was described as medium dense and containing sand. Measured index properties for select samples included water contents between 6.5 and 18.5 percent, dry unit weights between 101 and 108 pcf, and compressive strengths ranging between 2.3 and 3.4 ksf. Percent fines were determined to be 61 percent in one sample obtained from B-2.

The clay/silt layer was underlain by a silty sand layer at depths ranging between 10.5 (B-1) and 13 feet (B-2). This layer extended to the base of the borings (30 to 40 feet), and was generally characterized as medium dense to dense. Percent fines ranged between 27 and 35 percent and water contents between 3 and 16 percent. Three feet of low plastic sandy clay was present between 14.5 and 17.5 feet in B-1. Consistency of the layer was described as hard, as evident by an unconfined strength of 10.6 ksf in one sample. Water content and dry unit weight of the sample were measured at 12 percent and 113 pcf, respectively. The transition back to a silty sand below the clay was gradual as evident by the presence of a clayey sand between 17.5 and 23.5 feet. Percent fines for a select sample of the clayey sand were measured at 38 percent.

3.4.2 Groundwater Conditions

Groundwater level measurements were taken during and after completion of well installation. The piezometer readings taken both at the completion of well installation (April 20, 2001) and subsequently (July 30, 2001) indicated that there was no water down to a depth of 36 feet in boring B-1 and 30 feet in B-2. Furthermore, there were no indications of groundwater or wet soils noted during drilling and sampling activities. It is noted that the drilling operations were conducted in the summer. Groundwater levels and the presence of small localized perched water may change with seasonal fluctuations.

4.1 GENERAL

The proposed water conveyance pipeline is planned to be constructed using trenchless construction methods between Station 145+65 and 148+20. Tunneling would commence within a shaft at the east end and progress 255 feet to complete the crossing of I-215 and the BNSF railroad tracks. Profile design of the outside steel casing involves maintaining a near 0 percent grade with the crown elevation set at 1092 feet. Construction of pipeline segments other than the trenchless crossing will involve cut and cover methods.

This section of the report provides an evaluation of the following design and construction issues:

- Assessment of anticipated subsurface conditions to be encountered during construction;
- Evaluation of tunneling alternatives;
- Selection of a preferred tunneling approach, along with a discussion of key design and construction considerations for the approach; and
- Evaluation of key design and construction issues for the shaft excavations.

4.2 ANTICIPATED GROUND CONDITIONS

Tunneling operations are anticipated to encounter silty sands above the groundwater. Medium dense silt and very stiff clay were detected slightly above (silt) and below (clay) the pipeline horizon at the east end. The silty sand was characterized as medium dense to dense at the two boring locations. This material contained significant fines (30± percent) which will increase "stand-up" time experienced during tunneling. Considering the fines content, relative density and lack of groundwater, it is anticipated that the silty sand will exhibit slow raveling to firm behavior. Should loose zones of relatively clean sand be encountered, these soils would be expected to exhibit raveling behavior. Anticipated ground behavior as discussed herein is in accordance with the generalized categories of ground behavior for soft ground tunneling as summarized on Table 2. The profile on Figure 2 illustrates the anticipated subsurface conditions along the pipeline crossing.

Soils overlying the tunneling interval are primarily medium dense silty sands, medium dense silt and very stiff clays. Total overburden thickness over the tunnel interval will range between about 12 (east end) to 13 feet (west end). The crest of the I-215 embankment is positioned about 16 feet above the proposed pipeline.

As evident from lack of groundwater within the piezometers, groundwater is not anticipated within the required excavations. Piezometer readings were obtained during the spring and summer; hence, there could be some seasonal fluctuations in the groundwater level during wet seasons.

4.3 TRENCHLESS CONSTRUCTION ALTERNATIVES

Three trenchless construction methods were considered for the pipeline undercrossing. These options are microtunneling, pipe-jacking (with a shield) and bore-and-jack.

4.3.1 Microtunneling

Microtunneling is an underground method of constructing pipelines using a sophisticated, remotely controlled, laser guided, steerable boring machine. The pipe is installed by pipe jacking, which involves pushing pipe sections through the ground with hydraulic jacks assembled in a jacking frame. The jacking operation is conducted within an open excavation (jacking pit) positioned at one end of the crossing. Excavation is achieved using a mictrotunneling machine positioned in front of the lead pipe section. As the machine advances, the pipe is pushed forward simultaneously to maintain the opening. After pushing a full pipe section into the ground, a new pipe section is lowered into the jacking pit with a crane and connected to the previous pipe section. The process is repeated until the machine reaches the receiving pit. Spoils removal is remotely operated with either slurry or auger casing, depending upon the selected machine.

4.3.2 Pipe Jacking

Pipe Jacking (with a shield) is different from microtunneling in several ways. Excavation is carried out manually or with a cutterhead within the shield. In pipe jacking, the machine operator and other personnel are required to perform much of the work at the tunnel heading and inside the pipe string. Therefore, the inside diameter of the jacked pipe should be at least 4-feet to allow personnel access to the tunnel heading. Pipe jacking shields can utilize laser guidance systems, which are capable of maintaining accurate control of line and grade.

4.3.3 Bore-and-Jack

Bore-and-jack methods involve installing a pipeline by pushing a string of pipes through the ground with large hydraulic jacks situated within a jacking pit located at either end of the crossing. Soil excavation is conducted at the advancing end of the pipe string using continuous flight augers that are powered by a horizontal boring machine. For this type of installation, an outer steel casing is typically installed and the carrier pipe is placed inside the casing.

4.3.4 Alternatives Evaluation

Several elements of this project dictate the selection of a relatively simple trenchless construction method. Favorable subsurface conditions are present in the form of relatively dense silty sands above the groundwater. Furthermore, the casing pipe is of relatively small diameter (32 inches) and the crossing length is only about 255 feet.

The advantages typically offered by microtunneling (i.e. excavation through difficult soils under groundwater) are not applicable for this project. Microtunneling also presents cost and schedule disadvantages (as compared with bore-and-jack) as follows:

- Higher construction costs due to high cost of mobilization and setup of a microtunneling machine; and
- Longer construction duration due to longer setup time for microtunneling equipment.

Pipe jacking (with a shield) does not present a desirable alternative at this crossing due to the need for an oversized tunnel opening to provide personnel access. Provisions for access would be warranted if obstructions were anticipated in fill and/or native materials at the pipeline horizon. Obstructions are not expected in the silty sand alluvium; hence, a 16 to 18-inch increase in tunnel diameter is not recommended. Additional costs are incurred to backfill the oversized void between the casing and carrier pipes. Furthermore, shield/pipe jacking with the oversized opening may result in increased surface settlements requiring pre and/or post construction ground improvement.

It is our opinion that bore-and-jack method is the most favorable construction alternative for this project. This method provides a simplified approach that will meet the project criteria for installation of a 32-inch pipe on a relatively flat grade. The operation would commence within a jacking pit at Station 148+20 and proceed west. A receiving pit would be established at Station 145+60. These pits will serve as an extension of the open-cut construction to the east and west. Provisions for a staging area at the eastern limits is preferred due to limited surface constraints and reduced disturbance to the surrounding residential neighborhood. In addition, tunneling from the east will give the contractor approximately 100 feet to refine his methods prior to encountering the I-215 embankment footprint. The overall staging area in the vicinity of the jacking pit should provide sufficient space for storage of several pipe segments, loading of trucks to dispose of spoils and for crane access to lower pipe segments and for spoils removal.

4.4 GROUNDWATER CONTROL

Groundwater control is not an anticipated requirement for this project. Data obtained during the exploratory phase indicated that groundwater was not present within at least 15 feet of the invert. Actual groundwater levels at the time of construction may fluctuate depending upon the season and rainfall quantities. Although groundwater levels are not expected to rise above the proposed invert level, the contractor should be prepared to address potential localized perched water conditions should construction be conducted within wet seasons.

4.5 SURFACE SETTLEMENT

Settlement is the primary source of damage to adjacent streets, utilities, and residences during trenchless construction. Settlement is caused by loss of ground at the tunnel heading and by closure of the overcut void around the pipe. Selection of appropriate tunneling equipment and methods will limit ground loss, although some minor ground losses and surface settlement are unavoidable.

Estimates of the amount of surface settlement that could occur due to bore-and-jack operations were made to evaluate the potential impact on I-215 and the BNSF railroad tracks. Empirical methods have been developed for estimating surface settlement due to soft ground tunneling by the study of observed settlements on past projects (Peck, 1969; Cording and Hansmire, 1975). The settlement pattern that typically develops above a soft ground tunnel is a trough-shaped depression resembling an inverted bell shaped curve with maximum settlement occurring above the tunnel centerline (Peck, 1969). Calculations were made for a 36-inch ID pipe (even though a 32-inch ID is proposed). This provides a conservative estimate of the anticipated settlement. Assuming that the pipe springline is positioned at an average depth of 14 feet below the ground surface, we estimate a maximum surface settlement ranging between ⅜ and ½-inch. Maximum settlements of the interstate pavement could be slightly less considering the presence of the 4-foot high embankment. The width of the settlement trough is predicted to be about 12 feet.

The assumptions for this analysis were as follows:

- Appropriate tunneling methods are implemented and good construction practices are followed to limit ground loss;
- Volume loss at the casing head is between 0.5 and 1% of the excavated volume;
- A 3/8" thick casing band is used all around the casing, except at the bottom quadrant;
- A gap created by the casing band is not filled; hence, the entire 3/8-inch thickness contributes to volume loss; and
- Volume loss at the surface is 100% of the total volume loss at the tunnel.

Estimated settlement magnitude and extent are not expected to result in damage to the highway, railroad tracks and/or existing utilities. The Contractor should be responsible for repairing any damage resulting from tunneling or other construction operations. Although it is unlikely that damage will result from this small amount of settlement, pre-construction and post-construction inspection surveys should be completed along the pipeline alignment. Surface settlement points should be established along the alignment and within the theoretical settlement trough limits to monitor settlement during tunneling operations.

4.6 DISPOSAL OF EXCAVATED MATERIALS

It is assumed that materials excavated during tunnel construction can be stockpiled on-site for future backfilling of the pipeline excavation. Should excess materials be available, the soils can be hauled off-site for use or disposal elsewhere.

4.7 PRELIMINARY ASSESSMENT OF LIQUEFACTION POTENTIAL

Liquefaction is a soil behavior phenomenon in which a loose, saturated granular soil undergoes a rapid loss in strength due to the development of high excess pore water pressure generated by strong earthquake ground shaking. During earthquakes, liquefaction may result in ground failure and settlement with potentially severe damage to man-made structures. The piezometer readings at the site indicated that the water table was located well below the pipe invert. There is significant overburden pressure above the water table, thus liquefaction is not anticipated to occur at this location.

4.8 JACKING AND RECEIVING PITS

Excavations ranging between about 16 and 18 feet deep will be required for the jacking and receiving pits. These excavations will require shoring and excavation support to conduct work safely, protect surrounding site features and limit the area impacted by construction. Design and construction considerations for these excavations are discussed within this section.

4.8.1 Excavation

Excavation for the jacking and receiving pits are expected to encounter silty sand fill and alluvium, consisting of interbedded silts and clays overlying silty sand. The sands and silts are expected to be medium dense to dense, and the clays are anticipated to be stiff. Groundwater is not anticipated to be encountered within the excavations. Localized seeps, attributed to perched water may be present should construction occur during wet periods. It is anticipated that soils can be excavated using conventional excavation equipment such as a backhoe.

Near vertical excavations are required to limit construction impacts to the surrounding areas. These excavations will require shoring with an appropriate excavation support system. Any necessary utility location should be performed in advance of installing the excavation support appropriate to the contractor's proposed method of working. A temporary storing system consisting of soldier piles placed in drilled holes and either timber or steel plate lagging is considered appropriate for these excavations. Other excavation support systems such as liner plates with circular steel ribs for bracing, and circular steel ribs with timber lagging are also considered feasible. Driven sheet piles and soldier piles may be precluded because of noise and vibration impacts to the residential areas. Due to the depth of the excavations, it is anticipated that internal bracing will be required. Selection and design of the excavation support systems for the excavations should be the contractor's responsibility.

Lateral earth pressures that develop behind the selected support system will be a function of the type of support system, installation procedures, depth of excavation, retained subsurface materials and the magnitude of surcharge loads on the ground surface adjacent to the excavation. Based on soil conditions encountered in the exploratory borings, lateral earth pressures recommended for temporary excavation support design are presented on Figure 3. Passive pressure resistance will be developed against soldier piles below the base of the excavation. For soldier piles, the passive pressure should be applied to a width no greater than twice the diameter of the pile. A minimum factor-of-safety of 1.5 should be applied to the estimated passive resistance. Passive resistance should be ignored within the upper 2 feet of penetration below the excavation base. Minimum toe penetration of soldier piles should be at least 5 feet below the base of the excavation. Deeper toe penetration may be required to achieve a properly designed shoring system. All excavation work should be conducted in accordance with Cal OSHA trench excavation safety guidelines, including specific requirements for pit construction, protection, barricades, traffic control, egress/exit, and personal safety equipment.

The size and shape of the excavations for the jacking and receiving pits should be determined by the contractor, subject to any limitations shown on the plans, to suit the tunneling equipment and construction methods selected for the work. Jacking pit geometries for bore-and-jack operations typically range between about 12 and 15 feet wide by 20 to 30+ feet long. The dimensions of the jacking pit should consider the standard 20-foot length for casing pipe and necessary allowance for clearance between the casing and pit floor. Furthermore, the base of the jacking pit should be aligned with the proposed casing grade. Considerations should be made to provide for a mud mat at the base of the excavations to provide for a stable working surface, and to maintain the integrity of the subgrade soil surface.

A bore-and-jack operation requires that a square and secure backstop be provided for the track pushplate. The thrust force for the entire bore is transferred through the track to the backstop. The backstop should be designed to withstand a minimum of 2 times the maximum thrust capacity of the machine being used. For this project, it is anticipated that a machine with thrust capacity of at least 300 kips would be required. This assumes that a bentonite slurry lubricant would be employed to reduce friction at the soil/casing interface.

Receiving pit dimensions will likely be controlled by the requirements for the cut-and-cover/inclined pipe construction at the west end. Access to the pipes at this location will be required to assist in centering/securing the carrier pipe and for grouting operations.

4.8.2 Groundwater Control

Static groundwater levels are anticipated to be present below the base of the access pit excavations. A sump and pump may be required to remove water derived from storm events if wet periods are experienced.

4.8.3 Disposal of Excavated Materials

Materials excavated during construction of the access shafts should be stockpiled for future backfilling operations. Should excess cut materials be available, off-site disposal may be required.

4.9 BACKFILLING

Upon installation of the 32-inch outer steel casing pipe, the 20-inch carrier pipe should be assembled within the casing, centered and properly secured. The annular space between the pipe and casing should be filled with grout or low-density cellular concrete (LDCC). The mix design should provide a material that offers good flowability and limited shrinkage, so that the void space is completely infilled and the grout remains in tight contact with the two pipes.

This section presents a brief summary of major findings and conclusions concerning the design and construction of the pipeline crossing under I-215 and the BNSF Railroad. Findings and conclusions were established from our interpretation of subsurface conditions and experience with similar projects, based on sound engineering practice and current precedent set for trenchless construction.

- The proposed water conveyance pipeline will be constructed using trenchless construction methods between Station 145+65 and 148+20. Tunneling will commence from a jacking pit at the east end and progress 255 feet to complete the crossing of I-215 and the BNSF railroad tracks. Construction will involve installation of a 32-inch diameter steel casing followed by a 20-inch diameter internal carrier pipe. The pipeline will maintain a near 0 percent grade with the outer steel casing crown set at Elevation 1092.
- Trenchless construction operations are anticipated to encounter medium dense to dense silty sands above the groundwater. Laboratory testing indicates that the silty sand contains significant fine content (30± percent). Silts and clays may be in near proximity or within the excavation along localized segments.
- Overburden soil cover will range between about 12 and 13 feet. Additional cover is present at the I-215 embankment. Cover soils are expected to involve silty sand fill and alluvium, consisting of stiff clays and medium dense to dense silty sand and silt.
- Based on a review of the subsurface data and available trenchless construction methods, we recommend that bore-and-jack methods be employed to complete the pipeline crossing. This method offers a simplified approach that will meet project criteria for installation of a 32-inch steel casing on a relatively flat grade.
- Groundwater control during the tunneling operations is not expected to be required for the project.
- Maximum surface settlements are estimated to range between ⅜ to ½-inch. The surface settlement trough is predicted to be about 12 feet wide at the ground surface. The estimated settlement magnitude and geometry are not expected to result in damage to the highway, railroad tracks and existing utilities. Pre and post construction surveys are recommended to confirm that surface settlement criteria are achieved. Surface settlement points should be established along the pipeline to monitor settlement during tunneling operations.
- Access to the tunneling operations will be achieved through jacking and receiving pits constructed at the east and west ends of the work. Excavation depths are expected to reach about 16 to 18 feet; hence, shoring and excavation support will be required to maintain near vertical soil cuts.
- It is recommended that the annulus between the outer steel casing and carrier pipe be backfilled with grout or LDCC.

The conclusions presented in this report are based on the assumption that the soil and geologic conditions do not deviate substantially from those encountered in the exploratory borings. If any variations are encountered during construction, the Geotechnical Engineer should be contacted so that supplementary recommendations can be made. If the construction plans are changed from those presently conceived, the Geotechnical Engineer should review the changes and make modifications to the original recommendations presented in the report in order to meet the project needs.

The conclusions presented in this report were developed with the standard of care commonly used as state of the practice in the profession. No other warranties are included, either expressed or implied, as to the professional advice included in this report.

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Tables

Table 1
Summary of Laboratory Test Data

Sample Information				USCS Group Symbol	In Situ Water Content, %	In Situ Dry Unit Weight, pcf	Sieve				Atterberg Limits			Unconfined Compressive Strength, psf
Boring Number	Sample Number	Depth, Feet	Elevation, Feet MSL				Gravel, %	Sand, %	<#200, %	<2µ, %	LL	PL	PI	
B-1	2-4	4.5-5	1099.2	CL	10.6	108								13680
B-1	3	6-7.5	1097.2	CL			0	47	53	18				
B-1	4-4	9.5-10	1094.3	ML	6.5	101								3380
B-1	5	11-12.5	1092.2	SM			0	73	27	4				
B-1	6-2	13.5-14	1090.2	SM	3.1	92								
B-1	6-4	14.5-15	1089.2	CL	12.2	113					38	21	17	10590
B-1	7	20-21.5	1083.2	SC	6.7				39					
B-2	3-3	6.5-7	1100.5	ML	23.0	102								3500
B-2	3-4	7-7.5	1100.0	CL	20.0	107								6090
B-2	4	8.5-10	1098.0	CL			0	35	65	22	33	19	14	
B-2	5-3	11.5-12	1095.5	ML	18.5	108	0	39	61	10				2770
B-2	6	13.5-15	1093.0	SM	13.5				35					
B-2	7-3	16.5-17	1090.5	SM	16.0	108								2190
B-2	8	20-21.5	1086.5	SM										

Note: The laboratory tests were performed in general accordance with the following standards:
Water Content – ASTM Test Method D2216
Dry Unit Weight – ASTM Test Method D2937
Grain Size Analysis by Mechanical Sieving – ASTM Test Method D422
Atterberg Limits – ASTM Test Method D4318
Unconfined Compressive Strength Test – ASTM Test Method D2166

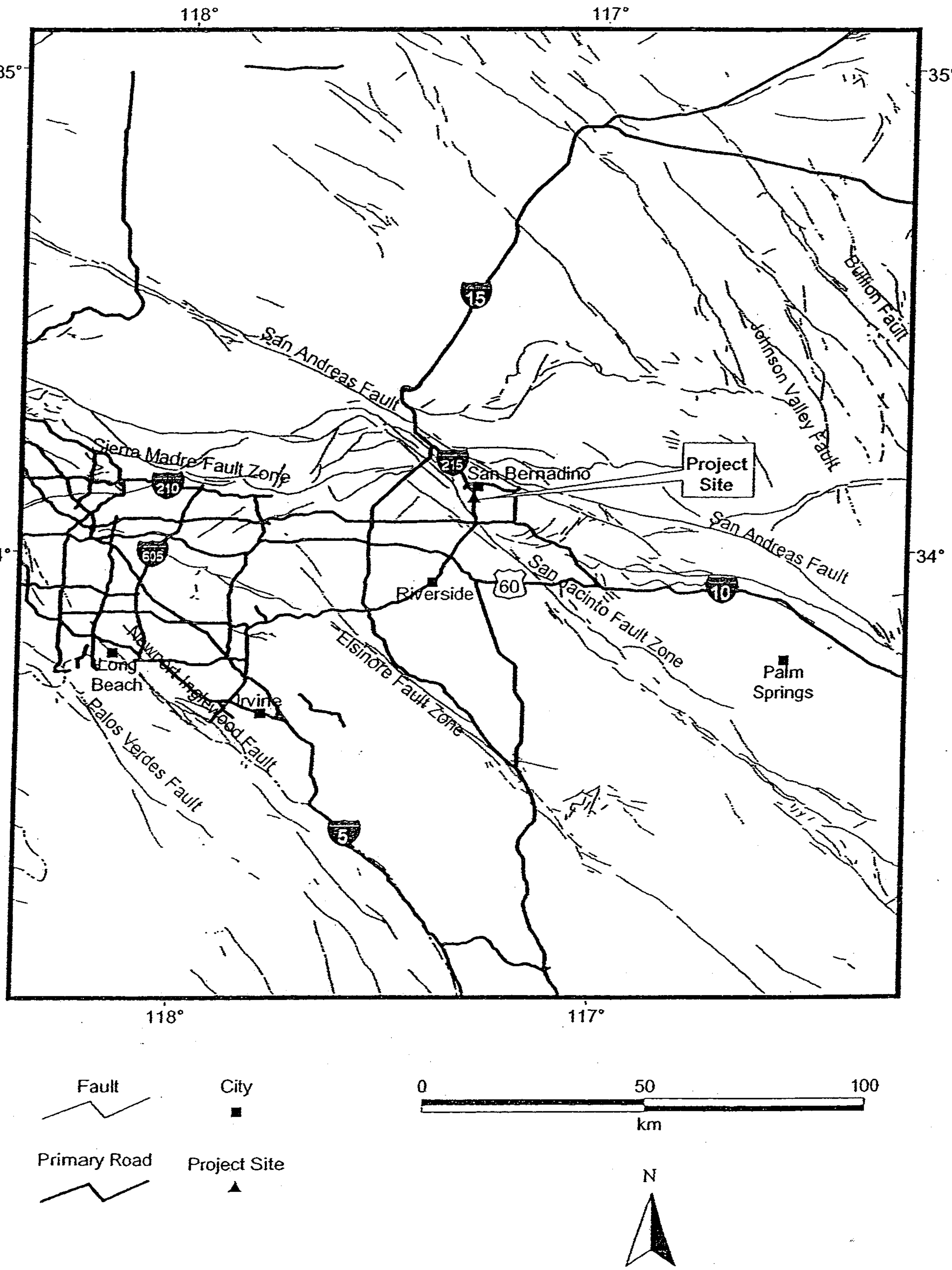
Table 2

Generalized Categories of Ground Behavior for Soft Ground Tunnels

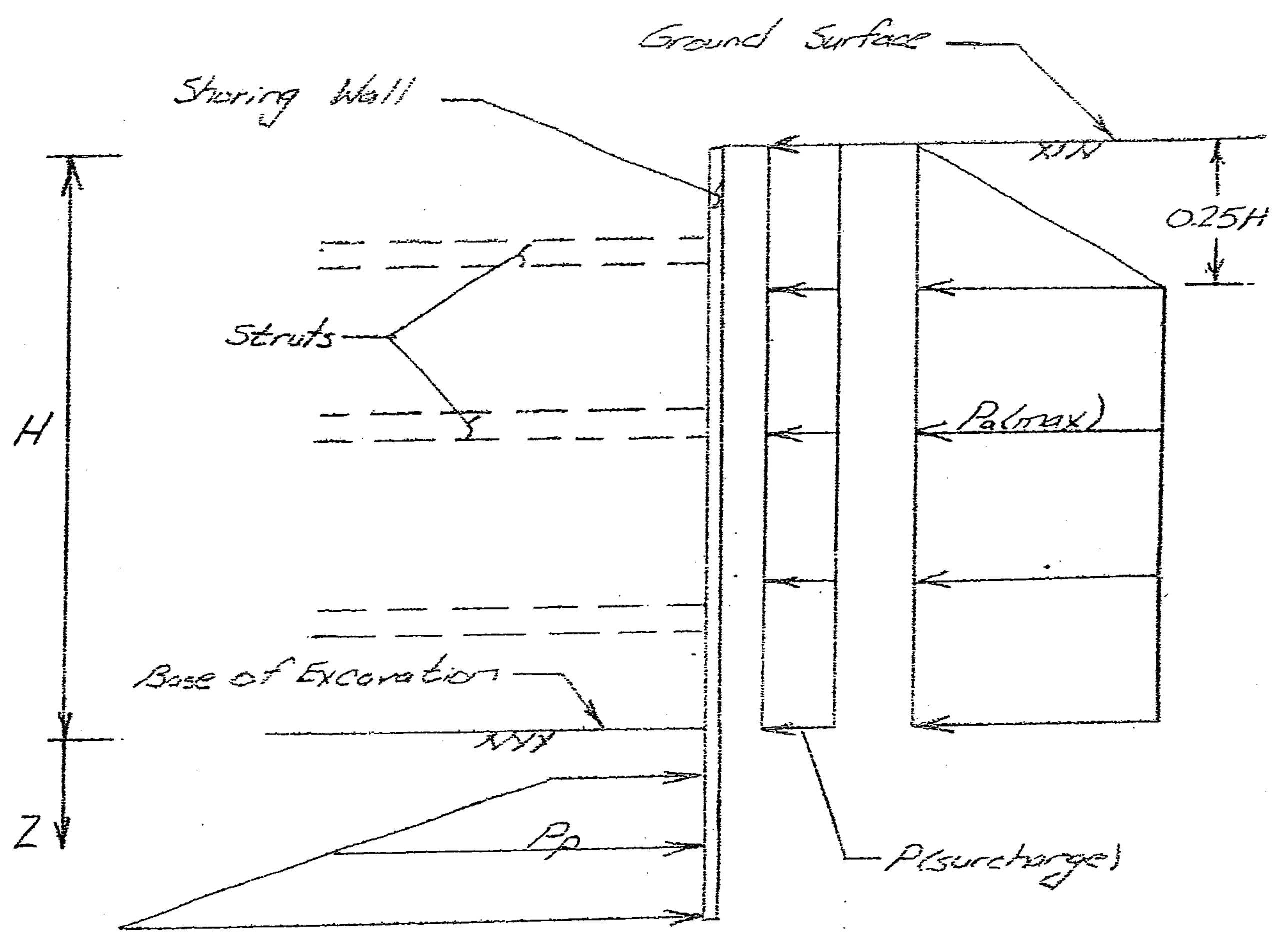
(After Terzaghi, 1950 as modified by Heuer, 1974)

GENERALIZED CATEGORY	GROUND BEHAVIOR
Firm Ground	A heading may be advanced several feet or more without immediate support. Hard clays, sands and gravels with clay binder, and cemented sand or gravel generally fall into this category.
Raveling Ground	After excavation, material above the tunnel or in the upper part of the working face tends to flake off and fall into the heading. In fast raveling ground, the process starts within a few minutes, otherwise the ground is slow raveling. Slightly cohesive sands, silts, and fine sands gaining thier strength from apparent cohesion typically exhibit this type of behavior.
Running Ground	Cohesionless, dry soils run from any unsupported vertical face until a stable slope forms at the natural angle of repose (i.e. approximately 30 degrees to 35 degrees). Running ground consists of dry, cohesionless materials, such as clean loose sand or gravel. Materials intermediate between running and raveling are described as cohesive running.
Flowing Ground	If seepage develops at the working face, raveling or running ground is transformed to flowing ground, which adavances like a viscous fluid into the heading. Silt, sand, or gravel below the water table without a high enough clay content to develop signficant cohesion will be flowing-type soils.
Swelling Ground	A condition where the ground absorbs water, increases in volume and expands slowly into the tunnel. This may occur in highly overconsolidated clays that exhibit high volume change characteristics upon wetting.
Squeezing Ground	Squeezing ground conditions are analagous to plastic flow, and the soil is observed to advance slowly into the tunnel excavation without any signs of fracturing. Squeezing occurs without an increase in the water content or a volume change in the soil and is governed by the soil strength in comparison to overburden pressure. Squeezing ground may include soft to medium stiff or stiff clays depending on the overburden pressure at the tunnel depth.

Figures



URS	Project No. 41-F2501400.11 Newmark Muscoy Plume Intermediate Design	REGIONAL QUATERNARY FAULTS	Figure 1



H	= Height of the Wall (feet)
Pa(max)	= Maximum Apparent Earth Pressure
	= (35 X H) psf
P(surcharge)	= 120 psf
Pp	= Ultimate Passive Pressure Along Embedded Section
	= (350 X Z) psf

Notes:

1. Horizontal surcharge pressure is based on a uniform adjacent vertical surcharge of 300 psf. This surcharge pressure may be insufficient and should be increased if large equipment or stockpiled soil will be immediately adjacent to the shoring.

2. A minimum factor-of-safety of 1.5 should be employed in passive earth pressure calculations.

URS	500 12th Street, Suite 200 Oakland, California tel: 510-893-3600 fax: 510-874-3268	LATERAL EARTH PRESSURE DIAGRAM TEMPORARY EXCAVATION SUPPORT I-215/BNSF UNDERCROSSING SAN BERNARDINO, CALIFORNIA	PROJECT NO: 41-F25014.00 DATE: 5-Aug-02	CHK'D BY: LTW FIGURE No. 3

Appendix A
Field Exploration Program

A.1 FIELD EXPLORATION PROGRAM

Two exploratory borings have been drilled as part of our field exploration program for the I-215 and BNSF tunnel undercrossing. The borings, designated B-1 and B-2, were drilled on the east and west side of the I-215 freeway, respectively. The approximate locations of the borings are shown on Figure 2. Boring B-1 was drilled at the location of the proposed jacking pit at the corner of Temple and 10th Street and B-2 was drilled at the location of the proposed receiving pit located at the corner of 10th and I Street. Borings B-1 and B-2 were drilled to a depth of 40 feet and 30.5 feet, respectively using a Mobil B-61 drill rig. Hollow-stem augers (8-in ID) were used to advance the entire length of the borings. The borings were performed by Cal Pac Drilling under the observation of Mr. Bill Gookin of URS's Santa Ana office. The drilling was performed on April 20, 2001.

Soil samples were obtained using modified California and Standard Penetration samplers driven with a 140-pound down-hole hammer falling 30 inches. The samplers were driven 18 inches into the soil and the blow count was recorded for each of three 6-inch increments. Samples were taken a 2.5-foot intervals in the upper 15 feet of each boring and then at 5-foot intervals to the bottom of the boring. Preliminary visual soil classifications were made in accordance with the Unified Soil Classification system. All recovered samples were screened for potential hydrocarbon contamination in the field using a Model 580B Organic Vapor monitor (OVM). The soil samples were screened by placing a portion of the sample in a zip-lock baggie, allowing it to sit for a few minutes and then inserting the OVM probe into the baggie to measure the vapors. Results of the OVM readings are shown on the boring logs in this appendix. No organic vapors were detected from the recovered samples in Borings B-1 and B-2.

The samples were collected using modified California sampler (2-inch I.D., 2-1/2-inch O.D.) and a Standard Penetration Test sampler (1-1/2-inch I.D., 2-inch O.D.). After advancing the samplers to the desired depth, the samplers were withdrawn from the borehole and the exposed soil was examined and classified. The modified California sampler was fitted with brass liners to contain the individual samples, which were sealed with plastic caps to preserve the natural moisture content. Samples recovered from the Standard Penetration Test sampler were sealed in a zip-lock bag to preserve the natural moisture content.

A.2 OBSERVATION WELL CONSTRUCTION

Groundwater observation wells were installed in Borings B-1 and B-2. The monitoring wells consisted of 10 to 15 feet of 2-inch diameter Schedule 40 slotted PVC casing (0.02-inch slots) overlain by 20 to 21 feet of unslotted PVC casing. The sand pack along the screened interval (and for 2 to 3 feet above the screened interval) consisted of #12 filter sand. The 1-foot seal above the sand was constructed using hydrated granular bentonite and the remainder of the annular space was backfilled with hydrated bentonite chips. Spot calculations were made to ensure that the volume of sand and bentonite matched the theoretical hole volume. Finally, a steel "Christy" box was installed flush with the road surface and fixed in place with concrete. The specific details of the observation well construction are shown on the boring logs. The borings and monitoring wells were completed under permit with the San Bernardino County Health Services Department.

Project: Muscoy Pipeline **Project Location:** San Bernardino, California **Project Number:** 41-F2501400.03	**Key to Log of Boring** Sheet 1 of 1

Elevation, feet	Depth, feet	SAMPLES Type	SAMPLES Number	SAMPLES Sampling Resistance	Graphic Log	MATERIAL DESCRIPTION	WELL SCHEMATIC AND CONSTRUCTION DETAILS	Water Content, %	Dry Unit Weight, pcf	REMARKS AND OTHER TESTS
1	2	3	4	5	6	7	8	9	10	11

COLUMN DESCRIPTIONS

1 **Elevation:** Elevation in feet referenced to mean sea level (MSL) or site datum.

2 **Depth:** Depth in feet below the ground surface.

3 **Sample Type:** Type of soil sample collected at depth interval shown; sampler symbols are explained below.

4 **Sample Number:** Sample identification number.

5 **Sampling Resistance:** Number of blows to advance driven sampler 12 inches beyond the first 6-inch interval, or distance noted, using a 140-lb hammer with a 30-inch drop.

6 **Graphic Log:** Graphic depiction of subsurface material encountered; typical symbols are explained below.

7 **Material Description:** Description of material encountered; may include color, moisture, grain size, and density/consistency.

8 **Well Schematic and Details:** Schematic of well installation; materials are described in the column to the right of the well schematic; graphic symbols are explained below.

9 **Water Content:** Water content of soil sample measured in laboratory, expressed as percentage of dry weight of specimen.

10 **Dry Unit Weight:** Dry density of soil sample measured in laboratory, in pounds per cubic foot (pcf).

11 **Remarks and Other Tests:** Comments about drilling, sampling, or well construction made by driller or field personnel. Also, field and laboratory test data using the following abbreviations:

HD	Hydrometer analysis, percent finer than 2 microns
LL	Liquid Limit (from Atterberg Limits), in percent
PI	Plasticity Index (from Atterberg Limits), in percent
PID	Photo-ionization device field screening, in ppm
SA	Sieve analysis, percent passing #200 sieve
WA	Wash on #200 sieve, percent passing #200 sieve
UC	Unconfined compressive strength test, Qu in psf

TYPICAL SOIL GRAPHIC SYMBOLS

- SAND (SP)
- SAND (SW)
- SAND with SILT (SP-SM)
- SILTY SAND (SM)
- LEAN CLAY (CL)
- FAT CLAY (CH)
- SILTY CLAY (CL)
- CLAYEY SAND (SC)
- SILT (ML)
- SILT (MH)
- SILT with CLAY (ML)
- GRAVEL (GP/GW)

TYPICAL WELL GRAPHIC SYMBOLS

- Blank casing in neat cement grout
- Blank casing in granular bentonite seal
- Blank casing in cement-bentonite grout
- Blank casing in filter sand
- Blank casing in hydrated bentonite chips
- Slotted casing in filter sand

TYPICAL SAMPLER GRAPHIC SYMBOLS

- Standard Penetration Test (SPT) unlined split spoon
- Shelby tube (thin-wall, fixed head)
- Modified California (2-1/2-inch OD) with brass liners
- Bulk or bucket
- California (3-inch OD) split barrel
- Grab from auger cuttings

OTHER GRAPHIC SYMBOLS

- First water encountered at time of drilling (ATD)
- Static water level measured in well on specified date
- Change in material properties within a lithologic stratum
- --- Inferred or transitional contact between lithologies

GENERAL NOTES

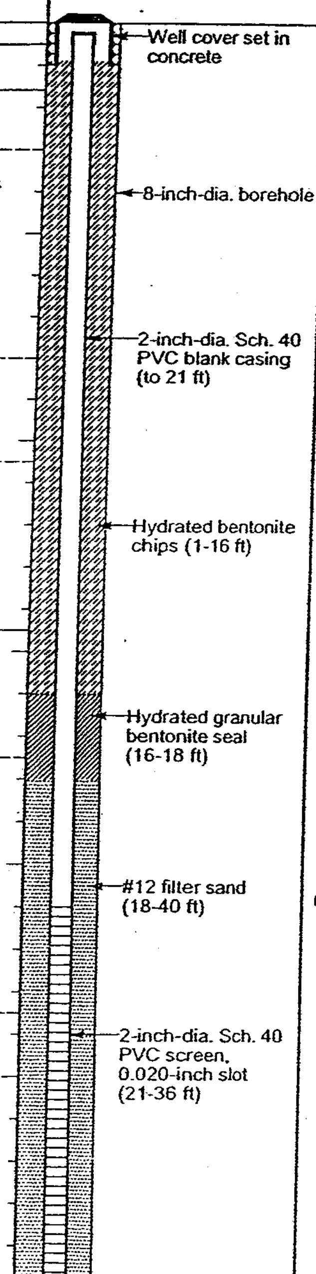
1. Soil classifications are based on the Unified Soil Classification System. Descriptions and stratum lines are interpretive; actual lithologic changes may be gradual. Field descriptions may have been modified to reflect results of lab tests.
2. Descriptions on these logs apply only at the specific boring locations and at the time the borings were advanced. They are not warranted to be representative of subsurface conditions at other locations or times.

Project: Muscoy Pipeline **Project Location:** San Bernardino, California **Project Number:** 41-F2501400.03	**Log of Boring B-1**

Date(s) Drilled	4/20/01	Logged By	B. Gookin	Reviewed By	M. Schmoll
Drilling Method	Hollow-Stem Auger	Drilling Contractor	Cal Pac Drilling	Total Depth of Borehole	40.0 feet
Drill Rig Type	Mobile B-61	Drill Bit Size/Type	8-inch-OD auger	Top of Casing Elevation	Not available
Sampling Method	SPT, Modified California, bulk	Hammer Data	Downhole; 140 lbs / 30-inch drop	Approximate Surface Elevation	1103.7 feet MSL
Water Level and Date Measured	Not encountered ATD	Borehole Completion	Piezometer installed; see schematic below for construction details	Location	Corner of Temple and 10th

Elevation, feet	Depth, feet	SAMPLES: Type	SAMPLES: Number	Sampling Resistance, blows / foot	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	REMARKS AND OTHER TESTS
	0				5 inches of asphalt concrete			Field screening of samples with PID Model 580B OVM. Background=0 ppm
					SILTY SAND (SM) [Fill] Medium dense, moist, medium brown			
			1	22	SILT (ML) and SILTY SAND (SM) [Fill] Medium dense, moist, dark brown, in alternating layers			PID=0 ppm
			SK-1					
1100			2	60	SANDY LEAN CLAY (CL) [Alluvium] Hard, dry to damp, light brown, with white calcite stringers			PID=0 ppm
	5					10.6	108	UC=13680 psf
			3	32				PID=0 ppm SA: 53%<#200 HD: 18%<2μ
1095			4	31	SILT (ML) [Alluvium] Medium dense, damp, light brown, trace sand, pinhole porosity observed			PID=0 ppm
	10					6.5	101	UC=3380 psf
			5	27	SILTY SAND (SM) [Alluvium] Medium dense, damp, light brown, weakly cemented to locally cohesionless			PID=0 ppm SA: 27%<#200 HD: 4%<2μ
1090			6	33		3.1	92	PID=0 ppm
	15				LEAN CLAY with SAND (CL) [Alluvium] Hard, moist, medium to dark brown	12.2	113	UC=10590 psf LL=38, PI=17
					CLAYEY SAND (SC) [Alluvium] Dense, moist, medium brown			
1085								
	20		7	35		6.7		PID=0 ppm WA=39%<#200
1080					SILTY SAND (SM) [Alluvium] Dense to very dense, moist, medium brown, friable, weakly cemented			
	25		8	70	Coarser-grained sand			PID=0 ppm
1075			9	50				PID=0 ppm
	30							

WELL SCHEMATIC AND CONSTRUCTION DETAILS



Report: GEO_12W_LAB_SNA; File: MUSCOYSB.GPJ; 5/8/2001 B-01

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Project: Muscoy Pipeline	Log of Boring B-1
Project Location: San Bernardino, California	Sheet 2 of 2
Project Number: 41-F2501400.03	

Elevation, feet	Depth, feet	SAMPLES Type	SAMPLES Number	SAMPLES Sampling Resistance, blows / foot	Graphic Log	MATERIAL DESCRIPTION	WELL SCHEMATIC AND CONSTRUCTION DETAILS	Water Content, %	Dry Unit Weight, pcf	REMARKS AND OTHER TESTS
	30					SILTY SAND (SM), dense to very dense, moist, medium brown, friable, weakly cemented (continued)	#12 filter sand (18-40 ft)			
1070							2-inch-dia. Sch. 40 PVC screen, 0.020-inch slot (21-36 ft)			
	35		10A							
			10B	19		SILT with SAND (ML) [Alluvium] Medium dense, moist, dark brown, micaceous				PID=0 ppm
						Becomes grayish brown with orange-brown staining				
1065			11	28						
	40					Bottom of boring at 40.0 feet				
1060										
	45									
1055										
	50									
1050										
	55									
1045										
	60									
1040										
	65									

Project: Muscoy Pipeline	**Log of Boring B-2**
Project Location: San Bernardino, California	Sheet 1 of 2
Project Number: 41-F2501400.03	

Date(s) Drilled	4/20/01	Logged By	B. Gookin	Reviewed By	M. Schmoll
Drilling Method	Hollow-Stem Auger	Drilling Contractor	Cal Pac Drilling	Total Depth of Borehole	30.5 feet
Drill Rig Type	Mobile B-61	Drill Bit Size/Type	8-inch-OD auger	Top of Casing Elevation	Not available
Sampling Method	SPT, Modified California, bulk	Hammer Data	Downhole; 140 lbs / 30-inch drop	Approximate Surface Elevation	1107 feet MSL
Water Level and Date Measured	Not encountered ATD	Borehole Completion	Piezometer installed; see schematic below for construction details	Location	Corner of 10th and I Street

Elevation, feet	Depth, feet	Samples: Number	Sampling Resistance, blows / foot	MATERIAL DESCRIPTION	WELL SCHEMATIC AND CONSTRUCTION DETAILS	Water Content, %	Dry Unit Weight, pcf	REMARKS AND OTHER TESTS
	0			4 inches of asphalt concrete	Well cover set in concrete			
		1A	26	SILTY SAND (SM) [Fill] Medium dense, moist, reddish brown				Field screening of samples with PID Model 580B OVM. Background=0 ppm PID=0 ppm
1105		1B						
		SK-2		SILT (ML) [Alluvium] Medium dense, moist, medium to dark brown, micaceous, with clay				
		2	25		8-inch-dia. borehole			No sample recovery; sample fell out in hole.
	5							
1100		3	31			23.0	102	PID=0 ppm UC=3500 psf
				SANDY LEAN CLAY (CL) [Alluvium] Very stiff, damp, medium brown, with white stringers	2-inch-dia. Sch. 40 PVC blank casing (to 20 ft)	20.0	107	UC=6090 psf
		4	24					PID=0 ppm SA: 65%<#200 HD: 22%<2μ LL=33, PI=14
	10							
				SANDY SILT (ML) [Alluvium] Medium dense, moist, brown and orange mottled, with clay				
1095		5	24		Hydrated bentonite chips (1-16 ft)	18.5	108	PID=0 ppm UC=2770 psf SA: 61%<#200 HD: 10%<2μ
				SILTY SAND (SM) [Alluvium] Medium dense, moist, brown and tan				
		6	24			13.5		PID=0 ppm WA=35%<#200
	15							
				Becomes brown				
1090		7	33		Hydrated granular bentonite seal (16-18 ft)	16.0	108	PID=0 ppm UC=2190 psf
	20							
		8	24		#12 filter sand (18-30.5 ft)			PID=0 ppm WA: 32%<#200
1085								
					2-inch-dia. Sch. 40 PVC screen, 0.020-inch slot (20-30 ft)			
	25			Becomes dense				
		9	50					PID=0 ppm
				Becomes tan				
		10A	36					
	30	10B		SILT (ML) [Alluvium], dense, moist, brown				PID=0 ppm

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Figure A-3

Project: Muscoy Pipeline Project Location: San Bernardino, California Project Number: 41-F2501400.03	Log of Boring B-2 Sheet 2 of 2

Elevation, feet	Depth, feet	SAMPLES Type	Number	Sampling Resistance, blows / foot	Graphic Log	MATERIAL DESCRIPTION	WELL SCHEMATIC AND CONSTRUCTION DETAILS	Water Content, %	Dry Unit Weight, pcf	REMARKS AND OTHER TESTS
	30					SILT (ML), dense, moist, brown (continued)				
						Bottom of boring at 30.5 feet				
1075										
	35									
1070										
	40									
1065										
	45									
1060										
	50									
1055										
	55									
1050										
	60									
1045										
	65									

Report: GEO_12W_LAB_SNA; File: MUSCOYSB.GPJ; 5/8/2001 B-02

Appendix B

Laboratory Testing Program

B.1 LABORATORY INVESTIGATION

This appendix presents the results of geotechnical laboratory tests completed as part of the investigation for the project. The geotechnical tests were performed on the recovered soil samples to help evaluate certain physical and mechanical properties, and to confirm the visual classifications made during our field investigation. Visual classifications of soils were completed per ASTM D2488, Standard Practice for Description and Identification of Soils. The geotechnical laboratory testing program included the following: moisture content, dry unit weight, grain size distribution and hydrometer, Atterberg limits and unconfined compressive strength. A summary of the test results is shown on Table 1 in the text of the report and on Table B-1 in this appendix.

The results of the Atterberg limits are shown on Figure B-1. The grain size distribution curves are shown on Figure B-2. Results of the moisture content, sieve No. 200 wash and unconfined compressive strength tests are shown on Table B-1 and on the boring logs in Appendix A.

The geotechnical tests were performed using the following ASTM standards.

Moisture Content	ASTM D2216
Dry Unit Weight	ASTM D2937
Grain Size Analysis	ASTM D422
Atterberg Limits	ASTM D4318
Unconfined Compressive Strength	ASTM D 2166